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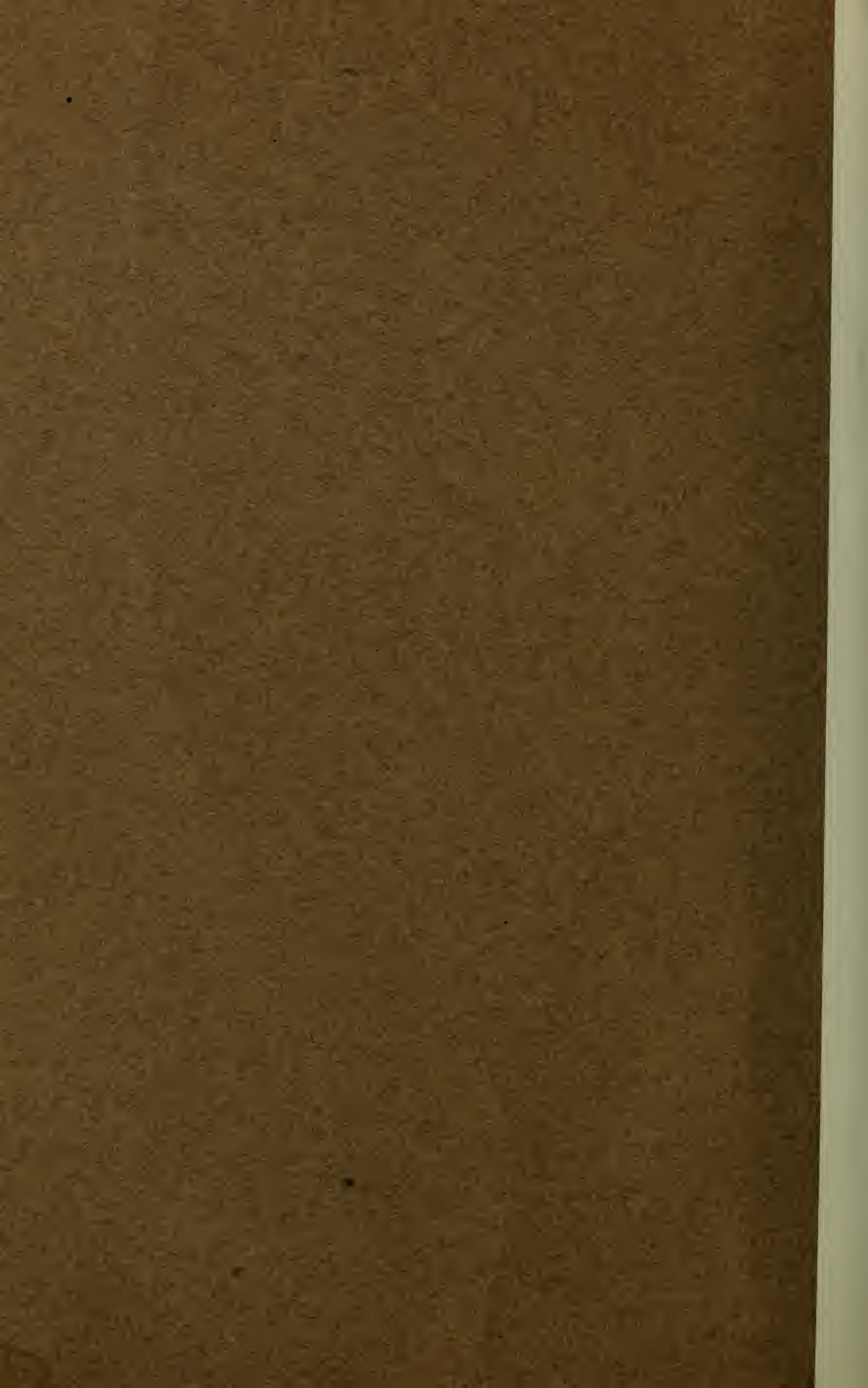
**Discussions on Papers and
Society Affairs**

Canadian Society of Civil Engineers
DECEMBER, 1908

Montreal :

**PRINTED FOR THE SOCIETY
BY THE WITNESS PRESS**

1908



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BUSINESS MEETING.

A business meeting of the Society was held on Thursday, October 8th, at 8.00 p.m., Mr. J. G. Legrand, M. Can. Soc. C. E., in the chair.

Messrs. W. Chase Thomson, F. P. Shearwood, and the Secretary were appointed Scrutineers for the Special Ballot for the election of Honorary Member, and declared Sir Sandford Fleming elected unanimously.

Messrs. J. G. Legrand, J. W. Heckman, N. F. Ballantyne, J. W. Orrock, and W. S. Deubelbeiss were appointed Scrutineers for the Ballot for the election of Members, and declared the following elected:

Members:

C. M. Canniff.

A. Jacquemart.

T. Turnbull.

Associate Members:

V. E. A. Belanger.

L. F. DeMiffonis.

G. G. Gale.

W. H. Munro.

F. Gaskell.

A. Nowlan.

M. L. Glover.

C. W. Noble.

W. K. Greenwood.

A. O'Meara.

E. E. Hawkins.

K. S. Patrick.

J. C. Holden.

P. M. Sauder.

A. R. Ketterson.

K. R. Schuster.

D. W. Krellwitz.

H. E. B. Smith.

H. MacNeil.

S. Wilkins.

Transferred to the Class of Member:

J. S. Costigan.

R. W. Macintyre.

R. R. Keely.

L. Sherwood.

Transferred to the Class of Associate Member:

E. C. Baker.

G. R. MacLeod.

P. Charton.

C. H. Mathewson.

J. H. Clarke.

E. E. Palmer.

E. P. Fetherstonhaugh.

C. W. P. Ramsay.

E. O. Fuce.

H. P. Renwick.

T. C. Irving, Jr.

J. Sears.

D. W. McLachlan.

Students:

W. D. Adams.

D. P. Bell-Irving.

J. B. Baird.

C. A. Bellisle.

D. R. Baribault.

H. H. Bliss.

Students:

H. A. Bowden.	H. A. Lawson.
A. O. Bourbonnais.	C. S. MacLean.
J. A. Brown.	C. Manseau.
G. A. Browne.	W. S. Malcolmson.
C. F. Carson.	D. W. McCrady.
F. Chappell.	S. J. McCuaig.
C. V. Christie.	W. T. McFarlane.
R. M. Christie.	F. H. McGuigan, Jr.
F. T. Cole.	F. H. McKechnie.
A. Cousineau.	J. E. Milne.
S. O. Cromie.	T. E. Morrison.
W. Dale.	F. O'Gara.
D. L. Derrom.	S. A. Ostrom.
O. T. Digby.	B. L. Parker.
H. H. Donnelly.	E. R. B. Pike.
L. O. R. Dozois.	C. E. Richardson.
R. H. Emmerson.	J. M. Robertson.
F. H. Emra.	J. C. Rondeau.
J. R. P. Estey.	C. C. Smith.
H. E. R. Hamilton.	R. S. Smith.
T. A. Hendry.	A. D. Stewart.
W. B. Hood.	A. Timbrell.
J. T. Johnston.	A. V. Tremaine.
D. E. Keeley.	M. T. Terrill.
J. A. Keefer.	N. E. Waddell.
L. A. Kenyon.	D. W. B. Walker.
A. Kerr.	A. C. Wright.
F. H. Kortright.	

MEETING OF GENERAL SECTION.

A meeting of the General Section was held Thursday evening, October 15, Mr. W. J. Francis, M. Can. Soc. C. E., in the chair. A paper, "Tests on Reinforced Concrete Beams," by Prof. E. Brown (A. M. Can. Soc. C. E.), was read by the author.

The discussion which took place immediately after the reading of the paper will be found in the report of the meeting held on November 19.

MEETING OF ELECTRICAL SECTION.

A meeting of the Electrical Section was held on Thursday evening, October 22, Mr. R. S. Kelsch, Chairman of the Section, in the chair.

A lecture on "Illumination Engineering" was given by Mr. V. R. Lansingh, Secretary of the Illumination Engineering

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Society, New York. The lecture was illustrated with a large number of lantern slides and blackboard sketches.

The manuscript of this address has not yet been received.

MEETING OF MECHANICAL SECTION.

A meeting of the Mechanical Section was held Thursday evening, October 29, Mr. W. H. Laurie, M. Can. Soc. C. E., in the chair. A paper, "Improvements at the Works of the Canadian Locomotive Co., Kingston, Ont.," by Mr. Henry Goldmark (M. Can. Soc. C. E.), was, in the author's absence, read by Mr. F. Gaskill, A. M. Can. Soc. C. E.

The following discussion took place:

MR. W. H. LAURIE (M. Can. Soc. C. E.)—The author has given a very good description of that part of the work with which the speaker was connected. Mr Laurie

It might be pointed out that the high-speed engine is a comparatively recent innovation in this country, but has for many years been deservedly popular in Great Britain and in other countries where it has been introduced. Ten or fifteen years ago, we never expected to get any economy out of a high-speed engine. The old Corliss was a slow-speed four-valve engine, and was looked upon as the standard of economy, but these new high-speed engines have proven themselves fully equal in point of economy, and have a great many points of superiority. Amongst these may be included the small amount of space they require, with the consequently reduced cost of foundations, the decreased radiation surface, and also the ease of accommodating their revolutions to those most suitable for the generator. Electrical generators are so largely used now that the quick-revolution engine is a valuable aid in reducing the initial cost of a combined installation. They also insure the fine governing necessary to proper regulation. Another point in favor of this engine is its high efficiency, which, according to the tests given in Mr. Goldmark's paper, is 92%, while in some larger plants it runs as high as 95%, which in the old days would have been considered phenomenal.

The secret of this is that the bearings are oiled by forced lubrication, so that there is a film of oil always between the working surfaces, and the efficiency, or rather the friction load, is practically the same through all the different loads the engine is called upon to carry. In other words, the friction loss on the light load does not increase with the full load in anything like the same ratio as with the slow-speed or in differently lubricated engines. That friction load is almost constant was proven by the late Professor Thurston in a series of very careful tests. The points

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Mr. Laurie brought to your attention serve to explain the facility with which these engines make long continuous runs without any adjustment being necessary or showing any appreciable wear.

Mr. Gaskill Mr. F. GASKILL (A. M. Can. Soc. C. E.) — With respect to this forced lubrication, are there different methods employed for superheated or saturated steam?

Mr. Laurie No. The forced lubrication has nothing whatever to do with superheated steam. The superheated steam only acts upon the cylinders, and these have a separate lubricating device to ensure lubrication. The forced lubrication is only used in connection with the bearings of the engine, and does not come in contact with the superheat at all. Of course it is necessary to use a very high flash lubricating compound for the cylinders under these conditions.

Mr. Ogilvy Mr. R. F. OGILVY (A. M. Can. Soc. C. E.)—Would you not look for different efficiencies in these two engines? One of them has twice the capacity of the other.

Mr. Laurie Usually a small engine has a slightly lower mechanical efficiency than a larger one, but this depends entirely upon conditions. The efficiency in all engines tends to improve after running for some time. On account of the forced lubrication and quick revolution of these engines, they can be very closely adjusted without causing undue friction and heating, and the consequently quiet running has a beneficial effect on the efficiency.

The speaker saw a paper recently, giving an account of a test made in the Old Country of one of these engines, which had been running three and a half years, almost continuously night and day. One year it ran 99.77% of the hours of the year, and after three and a half years the wear upon the bearings was found to be practically *nil*, owing to this system of forced lubrication.

Mr. Gaskill Could you give any idea of the back pressure on that engine with the exhaust steam used for heating purposes?

Mr. Laurie That depends upon the system of heating. Where the vacuum system is used, there is no back pressure at all. The speaker has seen such heating systems where the back pressure amounts to five and seven pounds.

Mr. Gaskill The question referred to this particular plant.

Mr. Laurie The speaker does not know what the back pressure is in this plant.

Mr. Gaskill If the engine is required to run non-condensing for heating purposes under full load, it would make quite a difference to the power. How do you cope with that?

Mr. Laurie The engines have sufficient margin to give the full power required, non-condensing, and in the larger sizes are provided with automatic expansion gear, which enables them to give the

most economical performance under varying conditions of load, Mr. Laurie steam pressure, back pressure, etc.

Mr. S. J. FISHER—Could you explain why it is that the smaller Mr. Fisher engine uses very much less water per indicated horse power hours than the larger engine does?

That would look as if an error had crept in there, somewhere. Mr. Laurie It should not be that way, but there must be some reason for it, because you see it is borne out through the tests with the half load condensing, and the half load non-condensing at different pressures. The smaller engine at half load non-condensing only takes 15.90, while the larger engine takes 16.90.

It may have been some of the units running in connection with it that caused the discrepancy. Of course, under similar conditions, the smaller engine would generally require a higher steam consumption than the larger one. With the full load conditions, the first test shows the large engine uses 13.94 and the small one 14.15, but even that is scarcely sufficient discrepancy.

The superheating is in favor of the smaller engine, which no Mr. Ogilvy doubt accounts for the apparent discrepancy.

That is so. And, of course, that would make quite a difference, Mr. Laurie because the 150 degrees superheat will make a difference of 15 to 18% in efficiency, which will account for a good deal of it. The superheat on the smaller engine is 194, which really accounts for the discrepancy. The superheat on the smaller engine is higher throughout the whole series of tests.

MINING SECTION MEETING.

A meeting of the Mining Section was held Thursday, November 12, Dr. J. B. Porter in the chair. A paper, "Modern Retort Coke Ovens, with Special Reference to the Practice of the Nova Scotia Steel and Coal Co.," by Mr. C. L. Cantley, Stud. Can. Soc. C. E., was read by the author and illustrated with lantern slides. The manuscript of the paper, which has not yet been received, will appear in the *Transactions*, together with the discussion which took place.

GENERAL SECTION MEETING.

A meeting of the General Section was held Thursday, November 19, Mr. Walter J. Francis in the chair. The following papers were read:

"Mean Sea Level at Quebec and New York," by W. Bell Dawson, M. Can. Soc. C. E.

"Notes on Canadian Forestry," by Stanislas Gagné, A. M. Can. Soc. C. E.

The following discussions took place:

DISCUSSION ON MR. GAGNÉ'S PAPER.

Mr. Francis

MR. W. J. FRANCIS (M. Can. Soc. C.E.)—It is to be regretted that Mr. Gagné was unable to be here to-night, as he would have been able to give us further information on this important matter in response to questions from members.

The Société is to be congratulated upon having this matter of forestry brought to its attention at this time. It is a very important subject, and is being taken up by many universities as a regular course of instruction. The United States authorities are beginning to realize the importance of the problem, and in May last President Roosevelt called a meeting of the Governors of the various States and the representatives of leading scientific societies, amongst whom was the President of the American Society of Civil Engineers. This meeting was held at Washington, and a committee was formed to study and report upon the conservation of the forests and natural resources of the United States.

The effects of forest destruction are very far reaching. It is believed by many that the continual troubles due to the erratic flow of the Mississippi river can be overcome by proper regulation of the headwaters, through re-forestation.

There is no doubt that Canada will follow this lead in due time. The Dominion has been called by Englishmen a "Wooden" country, but our people do not properly appreciate the importance of their heritage in forest resources, and the serious results that must follow a denudation of the land. Already the effects are being felt in Ontario, especially on the Grand River and the Thames, where the result of the total clearing of the forests in order to obtain the land for agricultural purposes is shown very plainly in roaring torrents in spring, which cause great havoc, and comparatively small streams later on. This matter has already been taken up, and efforts are being made by members of our Society to have these errors rectified.

WRITTEN DISCUSSION ON MR. GAGNE'S PAPER ON "FORESTRY."

By Mr. Henry Holgate (M.Can.Soc.C.E.).

Mr. Holgate

The subject of Mr. Gagne's paper is timely, is of present great importance, and demands the earnest attention of the Society.

The progress made in Forestry in Canada has not been very great, and the general organization is as yet not as effective as the importance of the work demands.

Canada was in times past a "wooden country," and it is popularly believed to be so now, but the cutting of timber has been carried on in such a relentless manner during the past (and this

still continues), that we will soon be brought face to face with the Mr. Holgate fact that we are poor in timber, and will become dependent on other countries for part of our supply.

Timber lands are part of Canada's natural resources, and should rightly accrue to the revenue of the nation. These lands are under the control of the respective Governments, and are administered by them. The desire to develop these resources has led to the disposal of vast areas of timber lands to individuals, who have exploited them, with no other restrictions than the paying of certain dues, which are in no way an equivalent of the value of the timber removed, and the evidence lies in the large profits realized by those who have succeeded in securing such timber lands. The writer would not say that this policy has not benefited the country in some respects, but as "hindsight is always better than foresight," it is quite plain where this policy leads to, and the country is to-day face to face with a serious condition which must be met, studied and settled, by adopting a policy for the future which will ensure safety to the timber supply.

When it is considered that in pulp wood alone Canada exports annually an equivalent of the annual product of 12 millions of acres of timber land, and that the supplying of its own railways with ties consumes the annual product of eight millions of acres of timber land, we must be convinced of the magnitude of the question.

This, together with the vast quantities of pine and other timbers used in Canada and exported, brings the total to figures which ought to cause us to pause, and "take stock" of what we really have, before dealing out periodically the timber areas in blocks measured by the square mile, to be slaughtered for the market, or to be held by the speculator.

It is probable that "political reasons" will guide the timber lands policy in some parts of Canada for some time, but a sound business policy surely must prevail eventually, and it is to be hoped that those in charge of such matters will early realize the national importance of the care of the forests entrusted to them.

The subject of Forestry is one, the study of which is forced upon us by conditions which arise. These conditions arose years ago in older countries, and consequently the study of them is in a more advanced stage than it is in Canada, and we have the great benefit of their experience.

The teaching of Forestry should be encouraged in our colleges and universities for the same reason that mining is taught. In 1906 in the United States, numerous universities had established Forest Schools, there were thirty State Forestry Associations,

Mr. Holgate besides the National Forest Department, with a Forester in each of twenty-six States. The importance and value of Forestry as practised in the United States is becoming greater every year, and is now organized upon lines which will ensure great National benefits.

It is very encouraging to see that the University of Toronto has established a course of instruction in Forestry, under the direction of Dr. B. E. Fernow, and this cannot but result in benefits to the Province of Ontario and the whole Dominion.

The organization of the Forestry Division of the Department of the Interior is in its embryo stage, but the writer believes that the Superintendent of Forestry, Mr. R. H. Campbell, fully realizes the importance of the work. This should be assisted and encouraged by the Dominion Government in a liberal manner, and an organization should be perfected which can properly administer the timber lands, by first gathering information regarding them which is now lacking, and which will enable the Government to deal intelligently with values. The Provinces also should at this stage take steps to improve their position in regard to the preservation of the timber and of perpetuating the supply.

In thirteen years the value of timber has increased 90%, due to the laws of supply and demand. Timber values are tending to increase, and the local as well as the foreign demand is on the increase, the supply is not increasing, and it is not good business to permit this latter condition to continue, when we have the means to improve it.

Canada's forest area is not known with accuracy; we may have 200 millions of acres of good timber land, much of it too far north to rapidly reproduce itself. There are other vast areas, however, which are well suited to the growth of timber, and which can be utilized for this purpose. Owing to our northern latitude it must be remembered that tree growth is not so rapid as in countries more favourably situated, and this is all the more reason for prosecuting the work of re-forestation at an early date, and devoting large areas to this purpose.

At the present time it would seem to be the best policy of the various Governments to:

1st. Perfect the Forestry Departments on a good scientific basis, employing properly qualified men for the work in all its branches.

2nd. To restrict the sale of timber lands, and forego the temptation of large gross revenue from their sales.

3rd. To devote a certain percentage of the net revenue of timber lands directly to re-planting, or in other words, reinvest a proportion of the profits in the business, in order to strengthen it.

Nature is kind to us, she gave us our stock-in-trade, but we are Mr. Holgate
destroying it wantonly. All that is necessary is to assist natural
processes, which we can do now practically without cost, and so
ensure our continuance in the timber business as a nation, and if
we do not take advantage of our potentialities, we deserve to suffer
the serious consequences which must follow.

The writer would earnestly ask the members of the Society to
consider this question of Forestry in its National sense, and to use
their influence in the direction of improving the conditions that
now exist, and of assisting in every way possible the advancement
of the study of Forestry, and the practical application of such
knowledge.

DISCUSSION OF PROF. BROWN'S PAPER, OCT. 15 AND NOV. 19.

MR. WALTER J. FRANCIS (M. Can. Soc. C. E.)—The Society is to Mr. Francis
be congratulated on having this interesting paper on such an
important subject as the testing of Reinforced Concrete. It is
gratifying to have a University man of Professor Brown's
standing devote so much time and careful work to these
researches. It is only by the co-operation of the laboratory
experts of our universities with the contracting engineers that we
can hope to arrive at a satisfactory working basis for this new
construction material. Professor Brown's paper shows that the
author appreciates very fully the difficulties under which the
constructor has to work, and in his discussion of the formulae he
eliminates all the theoretical and hair-splitting conditions which
possibly might be obtained in the laboratory if an unlimited amount
of time and money were available, but which cannot possibly be
obtained by the working constructor.

Only a few years ago we knew practically nothing of reinforced
concrete. Already we have a considerable amount of useful
information. By the combined co-operation of the laboratory
engineer and the constructing engineer we shall probably in a
few years know a great deal more about it.

In the course of the experience of the speaker he has had the
privilege of using hundreds of thousands of barrels of Portland
cement, and in the course of that experience he has made a great
number of tests, some of which have been quite satisfactory, but
others were, relatively speaking, dismal failures; and he has not
satisfied himself entirely that one can always foretell the result
of a test. This is undoubtedly due to our limited knowledge of
the materials. Such results are somewhat parallel to Professor
Brown's experience with that remarkable Ransome beam without

Mr. Francis diagonal reinforcement which he has so fully described. In conducting a series of tests it often happens that one or two specimens stand out pre-eminently for their good qualities after having been made under conditions apparently identical with all the other specimens. Why this should be so in the hands of a trained and experienced operator can probably be attributed to our present incomplete knowledge of the material. If such results as that Ransome beam can be obtained at rare intervals, then one is almost inclined to feel that there is some way of always obtaining them, and the speaker believes that the careful studies being made both in the testing room and outside of it will finally result in that end.

As the discussion proceeds the speaker would like to know what kind of sand was used in making the concrete; that is particularly important in the case of the roasted beams. If there were any calcareous matter in the sand it would have an injurious effect on the concrete at high temperature.

Speaking of the steel of the roasted beams carrying the heat into the interior of the concrete; would not the beams be a long enough time in the furnace to take up all the heat? It would seem that the rods would be nearly as hot as the furnace temperature, at any rate. Now, at about 1250 degrees F. the metal would be red hot. It is very interesting that such a condition does not entirely destroy the adhesion of the metal and concrete.

Mr. Morssen MR. M. MORSEN — In manufacturing reinforced concrete, the kind of reinforcement used should be entirely a question of cost; the Kahn, Johnson, and Ransome bars cost more than plain round bars. Did the author have any reason for testing such bars instead of plain ones?

Prof. Brown There was no special reason for making tests of these special bars only. The plain rod is often used, and various contractors who are using the plain rods have offered materials to the laboratory to make tests of beams made with that type of reinforcement. A great deal is heard of the value of the corrugated rod and the bond obtained between the rod and the cement, but the Hennebique Company are using plain rods for reinforcement. The Ransome system uses a twisted rod which it is claimed increases the yield point of the steel.

In no case were the beams damaged in roasting, except that the Kahn beams had cracks radiating from the wings. Nearly all the beams had a camber, and therefore had an initial strain before putting them in the testing machine. In one case the beam seemed to be yielding and then these cracks closed up, and we could load the beam up again. Local river sand (siliceous) was used.

For reinforcement, plain round rods can be bought in the open market at \$1.75, and fabricated on the ground for 50 cents per 100 lbs. If sufficient strength of beam can be obtained by using them there does not seem to be anything gained by using the more expensive shapes. Mr. Francis

WRITTEN DISCUSSION.

By Mr. F. P. Shearwood (M. Can. Soc. C. E.)

Mr. Brown's able experimenting and explicit arguments should do much to remove the complicated, unnecessary, and erroneous formulae used in designing reinforced concrete. In spite of the fact that there are so many and complicated theories made use of by designers for guidance in proportioning their work, it is probable that practical experience and instinctive judgment play the greater part in designing reinforced concrete work. It is this fact, as well as force of circumstances, that has in reinforced concrete beam work universally permitted the use of continuous construction. Mr. Shearwood

The general success attending monolithic construction with such brittle material as concrete (a material having practically no strength after passing its elastic limit) would seem to imply that it would be an advantage in structures built wholly of steel (a material which can distort greatly and retain its ultimate strength) to make use of the opportunities of full or partial continuity.

In connection with the continuity of reinforced beams, safer results may be obtained by using bars having a greater margin between the elastic limit and ultimate strength, and also a large ratio of surface to elastic limit so that if dangerous stresses are produced from any cause such as a settlement of the supports or internal strains, they can be adjusted by small distortions without too closely approaching the ultimate strength or adhesion of the concrete.

The loss in the value of the modulus of elasticity of concrete due to repeated loadings would undoubtedly influence the strain diagrams shown in the paper. It is unfortunate that more experiments treating with repeated loadings have not been made, for it is in the gradual breaking down of the adhesion of the concrete to the steel that failure is likely to occur. If the concrete in any of the beams described had absolutely uniform strength and modulus it would have broken up into an infinite number of slivers so minute that they could have no grip on the steel; in fact, it would have crumbled longitudinally. This does not occur because con-

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Mr. Shearwood concrete is not uniform in either strength or modulus, and so the straining is concentrated in a few cracks, enough being left intact to produce the desired connection to the steel. But is not this like counting on an uncertainty?

In the experiments described in the paper, especially those with Kahn bars, the partial action of the concrete in resisting tension, and the tension in the diagonal wings and unsheared portions of the bars, influence the strain diagrams, and it is probable that the useful tension of the concrete is far less than is shown by the extensometers.

In a paper read before the American Association of Civil Engineers, by Mr. Van Ornum, he gave an account of some instructive experiments which emphasized several dangers, and indicated how beams that have been constructed with great care will give erratic results when subjected to repeated loadings. He concluded from his tests that fatigue occurs when the loading is 50% of the ultimate, and that the ultimate may vary 30%. His experiments were not continued with loads below 50% of the ultimate, and his conclusions were arrived at by means of curves drawn from the results of the experiments. Practically all the beams tested broke after a varying number of repetitions, and it is questionable whether it is safe to assume that fatigue does not occur when the loads are less than 50% of the ultimate.

Railway bridges and other structures which support rapidly-applied loads, should be designed with a very liberal margin of safety; in fact, it cannot be theoretically shown that safety is permanent unless the strains in the concrete are kept within its tension value. This, of course, is an extreme view, and experience does no doubt justify higher unit stresses.

It may be that the better results obtained from beams having diagonal reinforcement are due to the fact that part of the steel is situated in the areas under compression or in those which have little or no tension, and therefore tensional cracks do not occur and destroy the adhesion. It is also this consideration that makes continuous construction so advisable, because it reverses the stresses in the flanges; and even if adhesion is destroyed where tension exists in the concrete, there is good connection in the compression areas.

While agreeing with Mr. Brown in his conclusions, it is likely that steel constructors sacrifice too much to mathematical safety, and can learn something from the success of reinforced concrete in utilizing much strength and stiffness which is capable of being derived with safety from conditions they now endeavour to avoid.

By Mr. Henry Holgate (M. Can. Soc. C. E.)

It is somewhat unfortunate that the test beams were not made in a manner more nearly approaching commercial conditions, instead of in the manner they were made, and consequently the results obtained will not be of such broad application as would be desired. Mr. Holgate

The proportions of the concrete should be given in volume or the equivalent of the proportions by weight should be given. The mixture used was fairly dry, whereas in beam work this practice is not followed, and instead of any tamping being done the process is more one of stirring in order to allow air bubbles to escape, and had the test beams been made of wet concrete, an equally smooth surface would have resulted.

Deformed reinforcement bars only were used, and as the larger amount of reinforced concrete is made with plain round rods, the results of these very careful tests are not quite as applicable to practice as they would have been had round rods been used as well as deformed ones. The writer has had such uniformly good results from the use of round rods that he has not had occasion once to use deformed bars for reinforcement.

It is hoped that in future experiments more attention will be paid to the more practical features of mixing and placing concrete so as to bring the tests nearer to working conditions.

Results obtained from careful designing of reinforced concrete are sometimes disappointing, and as an instance, the writer would mention a floor in Montreal which was designed for a live load of 250 pounds per square foot which was loaded with 1500 pounds per square foot and showed no effects from this overload. The overload was applied more than a year after construction, and the concrete had had ample time to mature, whereas the calculations were made as for concrete at 30 days. In most cases this would be an error, and if an economical design is to be attained, consideration must be given to the time which will elapse between construction and the date at which the work will receive its full load, and the work proportioned accordingly, so as to give credit to the concrete for its age when it is called upon to do the specified work.

In beam and column work, and especially in column work, the most effective reinforcement is cement; by that is meant that the larger the proportion of cement used, the better is the adhesion of the steel and concrete, and though this involves the frequent changing of proportion in concrete, it will pay to carefully proportion the mixture to the various parts of a structure rather than have an uniform mixture for several parts performing different

Mr. Holgate duties, and the amount of steel reinforcement used can be reduced to a minimum. The French engineers have long recognized this principle, and have produced some astounding results.

With regard to the heating tests, their practical application is resistance to the effects of fire, and in this connection it is most important to pay close attention to the composition of the aggregates. Generally speaking, concrete is a good fire resistant. The American Government supervised a series of fire tests on concrete lately. It was conclusively proved that the richer the concrete was in cement the less was the effect of the fire, and also that the concrete mixed with the greater percentage of water gave the best results, that is, the richer and wetter the mixture, the greater fire-resisting qualities had the concrete. It is almost needless to say that stone which will change its structure when subjected to 1700° F. should not be used for fire-resisting concrete.

Mr. Brown's paper presents a valuable addition to the literature on reinforced concrete, and it should be carefully studied by all members interested in such construction. He has succeeded in simplifying some of the process of calculation in a rational manner.

We can design for safety and economy, we can be certain of our materials, and we can, by exercising constant care, be sure of proper execution of the work, but "vigilance is the price of safety" pre-eminently in reinforced concrete work, and the most intelligent and painstaking supervision is necessary in construction if success is to be certain.

By H. M. MacKay (A. M. Can. Soc. C. E.)

Prof. MacKay In testing the reliability of the ordinary theory of flexure as applied to reinforced concrete beams it would be of interest to compare the calculated positions of the neutral line under small loads, before any cracks occur in the concrete, with those determined experimentally by the author. The writer understands that the author has some data covering this point, and hopes that he will present it in closing this discussion. A good agreement in this case would be a satisfactory check both upon the theory and upon the experimental work embodied in this paper.

It may be of interest also to trace the changes in the position of the neutral line, and in the values of the intensity of compressive stress in the outer layer of concrete under a given load as cracks progress from the tension side of the concrete towards the neutral line.

Using the author's notation, we may add the following symbols:

Let nd = depth from compression surface of concrete to point at which concrete ceases to carry tension, that is to the "top" of crack. It is evident that n must always be greater than x .

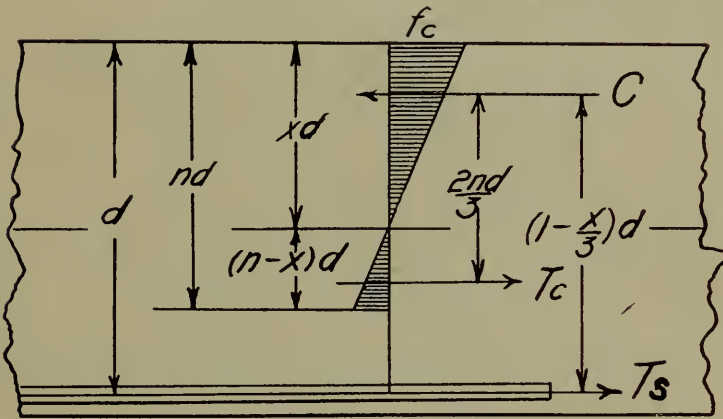
Let C = total compression in concrete.

Let T_c = total tension in concrete.

Let T_s = total tension in steel.

$$\text{Then } \frac{C}{T_c} = \left(\frac{x}{n-x} \right)^2 \quad \therefore T_c = \left(\frac{n-x}{x} \right)^2 C$$

$$\begin{aligned} T_s &= C - T_c = \left[1 - \left(\frac{n-x}{x} \right)^2 \right] C \\ &= \frac{2nx - n^2}{x^2} C \end{aligned}$$



Now taking moments of the tensile forces about the centre of compression we have

$$\begin{aligned} M &= \frac{2}{3} nd T_c + \left(1 - \frac{x}{3} \right) d T_s \\ &= \frac{2}{3} nd \left(\frac{n-x}{x} \right)^2 C + \left(1 - \frac{x}{3} \right) d \frac{2nx - n^2}{x^2} C \\ &= \frac{n(1-x)(2x-n) + 2x^2}{3x^2} C d \end{aligned}$$

$$\text{or } C = \frac{3 M x^2}{[n(1-x)(2x-n) + 2x^2] d}$$

$$\text{And } f_c = \frac{2C}{bx d}$$

As is shown elsewhere*

$$x = \frac{2pc + n^2}{2pc + 2n}$$

Prof. MacKay

By assigning different values to n in the above formulae we can trace the changes in the value of the intensity of compression stress in the outer layer of the concrete as the cracking of the concrete in the tension side progresses. Thus $n=1$ indicates that the concrete is uncracked above the centre of the steel reinforcement; $n=0.6$ indicates that a crack extends from the centre of reinforcement towards the neutral line a distance equal to $0.4d$, and so for other values.

The writer has applied the above analysis to the beam worked out by the author on page 69 of the paper. Assuming the beam to be carrying the full load for which it is designed changes in the value of intensity of compressive stress have been worked out for different values of n , that is for different conditions as regards the cracking of the concrete in tension.

The results are given in the following table:

n	x	C lbs.	Compression Area = bxd sq. ins.	$f_c = \frac{2C}{bxd}$ lbs./sq. in.
1.0	.52	37,540	157.1	478
0.9	.51	36,980	145.6	508
0.8	.47	35,880	134.2	535
0.7	.43	34,780	122.8	566
0.6	.40	33,120	114.2	580
0.5	.375	31,460	107.1	588

$p = .00675$ $c = 15$ $d = 20.4$ ins. $b = 14$ ins.
 $M = 552,000$ in. lbs.

Some of these values of n are higher than would be possible in reality. Thus $n=1$ would, under the assumed load, result in an intensity of tensile stress in the concrete of about 500 lbs. per sq. in., which is, of course, much above the tensile strength of ordinary concrete. The results indicate, nevertheless, that the intensity of compressive stress in the outer layer of concrete changes but little for a fixed load under different conditions of cracking, as the neutral axis adjusts itself automatically as the crack progresses.

The writer believes that the author would agree with him in doubting whether the high value of compressive stress obtained for the beam in question, under the conditions assumed on page 69, would actually be realized.

The writer considers that this paper is one of the clearest and most valuable contributions that has yet been made to the literature of reinforced concrete.

By Mr. M. Morssen.

Plain and reinforced concrete are not to be considered as Mr. Morssen materials which will resist fire at high temperature indefinitely—building materials of such qualities are unknown—but concrete, especially reinforced, is certainly a building material which will resist flames even at a high temperature during conflagration without losing its strength. If the results of laboratory fire tests are not always satisfactory, it is principally due to the difference of conditions. A small beam roasted in a stove for several hours is under quite different conditions from a beam of the same size forming part of a floor during a fire. The writer is quite confident that in the latter case the beam will resist a higher temperature and lose less of its strength.

Tests have shown that when exposed to heat, the temperature of the concrete beam will rise up to about 200 or 212° F. in a short time, and then will remain almost stationary as long as any water remains in the concrete, as the heat will be taken up by evaporation. Further, when roasted in a stove, there is no radiation possible, the beam being surrounded by heat on all sides. As a part of a building, however, free radiation and the surrounding humidity help greatly, and the temperature of the concrete does not rise so quickly though the heat of the conflagration be very great. The writer assisted at some interesting fire tests made in Paris in 1900, during the World's Fair.

A small building of reinforced concrete (gravel concrete) (1-4), was submitted to a fire test extending over several days. The temperature maintained was about 1000° C. (1832° F.), during two hours. The first day some concrete spalled off from small parts of the floor, the next day only the plastering was damaged. The top floor was eight cm. ($3\frac{1}{2}$ in. thick), and so little heat went through that one could bear the hand on it. During the fire tests the floor was loaded from 500 to 1000 kg. per square metre (about 200 to 400 lbs. per sq. ft.), and the deflection was as follows:

After 10 - 1mm. = .04 in.
28 - 7mm. = .25 in.
45 - 12mm. = .50 in.
60 - $13\frac{1}{2}$ mm. = .54 in.

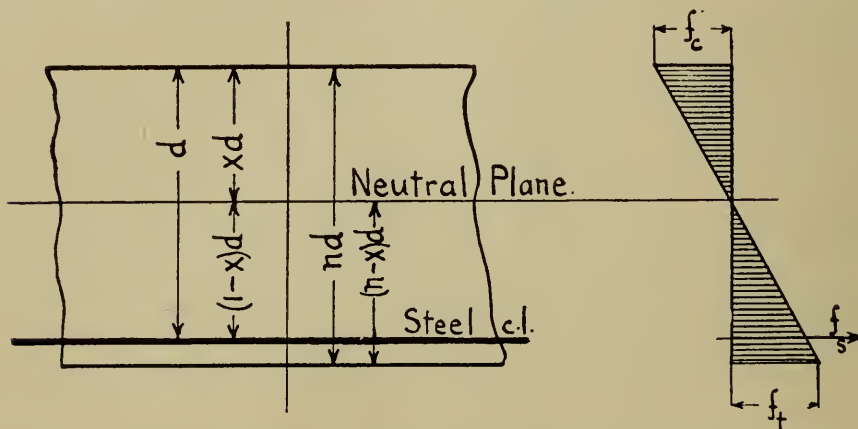
After one hour the whole building was flooded with water, and the deflection was reduced by about 30%, and when unloaded disappeared quite completely. These tests have proven that for practical purposes reinforced concrete is fireproof. Furthermore, some great conflagrations in Europe and in America have shown

Mr. Morssen its good fireproof qualities. The results of other laboratory tests have further proven that plain concrete when exposed to heat suffers more than when reinforced. The writer has read Professor Brown's paper with real pleasure. Tests and conclusion are represented in a clear, very comprehensive, and complete way, and one can only wish that this scientific investigation be continued and similar tests be made, not only for single beams of rectangular sections, but for beams of a T section, as this section is the most frequently encountered in the design of buildings. We have to bear in mind that there is a great difference between structures of reinforced concrete and structures of steel, wood, etc. The latter are composed of single elements bolted or rivetted together, and laboratory tests on the elements are more closely justified, but reinforced concrete structures are, so to speak, poured in one piece, and the difference between the laboratory elements and the actual case where a whole monolithic floor is tested may be great. Therefore, in conducting the tests, these conditions should be taken into consideration.

APPENDIX.

Prof. Brown Location of neutral line, allowing for tension in concrete.

Using the symbols explained on page 8 of the paper, and



assuming E_c for concrete to have the same value in tension and compression,

$$\frac{f_c}{f_t} = \frac{x}{(n-x)} = \frac{x}{n-x} \quad \text{i.e.} \quad f_t = f_c \frac{n-x}{x}$$

$$\text{Also } \frac{f_c}{f_s} = \frac{x}{(1-x)c} \quad \text{where } c = \frac{E_s}{E_c}$$

$$f_s = f_c \frac{(1-x)c}{x}$$

Then since the total tension is equal to the total compression, Prof. Brown

$$\begin{aligned} f_c \frac{bx d}{2} &= f_s p b d + f_t \frac{b(n-x)^2 d}{2} \\ &= f_c p b d \frac{(1-x)c}{x} + f_c \frac{b(n-x)^2 d}{2x} \end{aligned}$$

which gives

$$\begin{aligned} x^2 &= 2pc(1-x) + (n-x)^2 \\ x^2 &= 2pc - 2pcx + n^2 - 2nx + x^2 \end{aligned}$$

or

$$x = \frac{2pc + n^2}{2pc + 2n}$$

In applying this formula at the early stages of the loading at which the concrete is least likely to have any cracks, nd may be put equal to the full depth of beam, so that n is greater than unity, d being the depth to the steel reinforcement.

EXAMPLES.

(1) *Johnson Beam No. 5*, p. 28.

$$\begin{aligned} nd &= 11.75'' & n &= 1.07 & p &= 0.0102 \\ d &= 11.0'' & \text{Take } c &= 10, \text{ i.e., } E_c &= 3 \times 10^6 \text{ lb. per. sq. in.} \end{aligned}$$

$$\begin{aligned} \text{Then } x &= \frac{2 \times 10 \times 0.0102 + 1.07^2}{2 \times 10 \times 0.0102 + 2.14} \\ &= \frac{0.204 + 1.145}{0.204 + 2.14} = \frac{1.349}{2.344} = 0.575 \end{aligned}$$

The curve on p. 63 shows that x was initially about 0.525.

(2) *Ransome Beam, No. 14*, p. 44.

$$\begin{aligned} nd &= 9.5'' & n &= 1.086 & p &= 0.0127 \\ d &= 8.75'' & c &= 10 \end{aligned}$$

$$\begin{aligned} x &= \frac{20 \times 0.0127 + 1.086^2}{20 \times 0.0127 + 2.172} = \frac{0.254 + 1.18}{0.254 + 2.172} \\ &= \frac{1.434}{2.426} = 0.59 \end{aligned}$$

The curve on p. 66 shows that x was initially about 0.55.

Prof. Brown The agreement in the above cases between the theoretical and experimental position of the neutral axis is reasonably close. The value of E_c is not known to any great degree of accuracy, and if c is less than 10, the agreement would be closer.

The following table shows similar comparative values for other beams taken at random from the paper. The observed values recorded are taken from readings at lower loads than those shown in the plates accompanying the paper, as it is only at these loads that there can be any real feeling of certainty that cracking is practically absent. The agreement between theoretical and observed values will be seen to be very close.

Beam	Values of x	
	Theoretical	By Observation
Kahn, No. 6, p. 31	0.595	0.58
Kahn, No. 13, p. 42.....	0.585	0.56
Johnson, No. 3, p. 19.....	0.597	0.60

Mr. Francis The speaker had prepared a short written discussion covering very much the same ground as that taken by Mr. Holgate, raising the question as to how Professor Brown succeeded in making beams of what he calls a reasonably stiff mixture without departing materially from the ordinary method of construction of reinforced concrete, using the very wet mixture. The speaker also intended to quote from Professor Talbot in a recent *University of Illinois Bulletin*, No. 20, which article Mr. Holgate evidently had in mind.

In this *Bulletin* Professor Talbot, speaking of a long series of tests on reinforced concrete columns, says:

"The writer feels that there is yet much to be learned about concrete and reinforced concrete columns. After making the tests described, he has had to give up several preconceived notions of the action of hooped concrete, notions based upon meagre statements of tests and analyses made from assumed conditions. The tests here given are not altogether conclusive, but they may help to clear up uncertainties and give an idea of the action of concrete when restrained by hooping. The tests on plain concrete columns also bring out instructive results. The following observations on the tests may be made:

"1. The average ultimate strength of the plain concrete columns

at ages not far from sixty days is as follows: 1-1½-3 concrete, 2,300 lbs. per square inch; 1-2-4 concrete, 1,740 lbs. per square inch; 1-3-6 concrete, 1,030 lbs. per square inch; 1-4-8 concrete, 575 lbs. per square inch. The effect of the amount of cement used in the concrete is very marked, and cement is shown to be an economical reinforcing material for compression members. The average variation of individual results from the average strength in the case of 1-2-4 concrete about 60 days old is 12%, ranging from 34% below to 25% above the average. The tests made at an age of six months indicate an increase of something like 15% over the strength at 60 days." Mr. Francis

This article is quoted in order to point out how differently Professor Brown and Professor Talbot carried out their experiments. In his work Professor Brown has used only one mixture, the 1-2-4, and Professor Talbot proves cement itself to be one of the best and cheapest reinforcing materials. It would be interesting to have tests with other mixtures made if time and circumstances would permit.

We have with us to-night Mr. Gustave Kahn, of Toronto, who has prepared a written discussion, which I will now call upon Mr. Kahn to present.

MR. GUSTAVE KAHN—The writer received a copy of the paper read by Mr. Brown before the Society on October 15th, and naturally felt considerably disappointed, not so much on account of the reports of the tests in question themselves, but upon the conclusions drawn by the author. Feeling that an injustice had been done in drawing these conclusions, the writer addressed to the author a personal letter calling his attention to the points in the paper which appeared to the writer in error, and it is through the courtesy of Mr. Brown that the writer has the privilege of entering this discussion. Mr. G. Kahn

The writer does not wish to be understood for a single moment as wishing to throw any discredit upon the integrity of purpose in the making of these tests, but feels that there is a general agreement of there being a great difference between theoretical workings and the practical doings in reinforced concrete.

Mr. Brown calls attention in a number of instances in his paper to the fact that the Kahn bar was procured in accordance with the generally adopted or standard form in which these bars are manufactured, viz.: sheared to the centre. This is unquestionably an error, since in the standard form of Kahn bar the center third of the bar is left unsheared. (See Kahn argument in Handbook, p. 14). Since Mr. Brown's entire discussion of the Kahn bar is practically based on this assumption, it would seem that a thorough study

Mr. G. Kahn should first have been made upon the prevailing practice in the use of this bar.

The trussed bar is sheared in a great variety of ways. When it is used for concentrated loading it is invariably sheared closely up to the point of concentration of load. When used for uniformly distributed loading, it is always left unsheared for a portion of its length in the centre and the shear members are not struck up until a point has been reached where the bending moment is sufficiently reduced to warrant reduction in cross section of the beam.

When the beam is subjected to loading at the third point, the bar is generally sheared from one end to a distance very near the point of concentration of load, the centre being left unsheared.

It is very often the case that one end of the bar is sheared for a greater distance than the other end, and sometimes, as a matter of fact, where the beam is used in cantilevered construction combined with the ordinary construction, a portion of the bar has the shear members turned upwards, whereas the others are turned downwards.

It will therefore be realized that the trussed bar is applied and manufactured to suit the desired results.

In beam construction almost invariably more than one trussed bar is used. Some of these bars are sheared directly up to the centers and others are left unsheared for, say, one-third of the length of the entire bar, and still others for half of the entire bar, the unsheared portions being dependent upon the load in every case, the result being that the cross sectional area of the steel in tension is reduced pro rata with the reduction of the bending moment, and the shear members increased as the shear increases.

It will be granted at once that probably no structure could be so readily misused as the ordinary truss, and if the typical truss without modification were applied by the engineer to any and every problem he would certainly err most grossly. To assume that the manufacturers of the trussed bar manufactured only the one form would place them in a class with the man who in his ignorance uses the single truss for one and every need.

There is no question but that the trussed bar has been in many instances improperly used, in just the same manner as the truss itself, or, in fact, the same manner as any reinforcement may have been improperly used. No one will question that, with the ordinary Pratt truss any load can be carried, provided the members are properly proportioned for their respective loads, and further that no other form of truss can be adapted so universally. The trussed bar is nothing more or less than the tension member of the Pratt

truss. It must, of course, always be remembered that the beam Mr G. Kahn reinforced with trussed bars should be so designed as to have a sufficient number of these bars sheared in such a manner that this tension and shear are sufficiently provided for at every point.

Mr. Brown will no doubt therefore agree that the use of the trussed bar lends itself very well to design, and that the correct application in every case should be studied by the Engineer with just the same degree of care as he would use in studying his design were he to use plain or deformed straight line bars.

Mr. Brown will no doubt also grant that no commercially practical bar has been designed up to date that will accomplish the proper distribution of metal both in tension and shear, if only one bar be used in a beam, for in order to do this with one bar, assuming that the section used by the manufacturers of the trussed bar were used, it would be necessary to vary the shear members from the centre to the end—in other words it would be necessary to shear from the bar near the centre, a small fraction of the wings, at the next panel point (assuming that the beam or truss were divided into panels), a proportionately larger section of the wing would have to be sheared and struck up, and thus the shear members would have to increase in size as the end of the beam is approached. This result is actually obtained in the use of the trussed bar when the beam is reinforced with a number of bars. First, the smallest, or centre bar, as it is termed, is sheared to the centre, the next bar or set of bars are sheared to the third points, and the next bar or set of bars are sheared from the quarter points, so that at the end the shear members would greatly exceed the shear members at any other point in the beam.

Mr. Brown in his paper further says that when the trussed bar is used, fully one-third of the metal performs no other function than to take care of shear. You will undoubtedly see that this is in error, for when the bars are placed within the beam in a manner consistent with good engineering practice, less than 5 per cent. of the total metal in the beam is sheared up at the centre, and this amount is increased as the shear in the beam increases, until at the end of the beam one-third (or even more, if the case requires) of the total steel in the beam is used for shear reinforcement. In no case is it necessary in actual building construction to place a greater amount of material in the beam when the trussed bars are used than is actually needed in cross sectional area for bending moment. Thus it will be seen that the Kahn bar will lend itself most readily and absolutely correctly to the reinforcement of concrete beams when a number of bars are used, the shear reinforcement being varied in the proper manner.

Mr. G. Kahn There is this advantage with the trussed bar: it is not liable to be placed wrongly, the only responsibility resting upon the workman in charge is, that he place the bars sufficiently high or low in the concrete and that he bend the diagonals upward at 45 degrees. It must be borne in mind that the average building superintendent, or foreman in charge, is, or should be, a practical workman, and he should be relieved as much as possible from every scientific detail. His knowledge should be devoted to the proper construction of forms, the proper placing and mixing of concrete, as well as steel. The entire work should be so designed and planned by the engineers in charge that the proper assembling of the materials may be simplified as much as possible, and this is admirably accomplished by using the Kahn bar. There being no necessity to handle loose stirrups and practically no bending to be done in the field, the liability of errors creeping into the work is reduced to a minimum. If there were really a slight amount more steel necessary when using Kahn system than when using plain or ordinary deformed bar systems, and if nothing more were accomplished than that the liability of error creeping into the work were reduced, then the slight additional amount of steel used would be amply paid for.

The tests made by Mr. Brown are on beams loaded at the third points. It is very true that so far as bending moment alone is concerned, this method of loading approximates a uniform distribution of loading, but so far as shear is concerned, the effect of the two-point loading is only three-quarters of the intensity of the uniform load.

The trussed bar is fundamentally advantageous for beams of uniform loading, where its great shear resisting capacity comes fully into play. Since in the majority of the plain bar tests the beam failed by shear and not by bending moment, it would seem that their strength should be based on their shear resisting capacity and any calculations based upon the cross sectional area of the steel in the centre of the beam must be erroneous since the beams actually failed near the end.

The theoretical arrangement of steel for two-point loading is the trapezoidal form, and if the straight line bars were secured at their ends so as to develop the full tension of the bar at this point, there is no question but this bar should develop equally as good results with the trussed bar, provided of course that the properties of the steel were the same for both reinforcements.

Theoretically, however, the trussed bar should, even in this case, provide a much stiffer beam than the trapezoidally reinforced

beam, since the steel is mainly near the bottom, and the efficient depth is not decreased near the end of the beam. Mr. G. Kahn

If the trussed bar is used for two-point loading, its shear members should extend from the point of loading to the end of the beam, but the bar should remain unsheared in the center third. It is, of course, questionable whether the full cross section should be taken into account in this case, since the shear is maximum directly under the load. There is no advantage whatever in shearing the bars to the centre, and in the present instance the beams would actually have tested to the same point had the wings in the middle third of the beam been sheared completely from the bar. The shear members in the centre of the beams were entirely lost. Bars were readily procurable with the central portion unsheared. There is no reason why Mr. Brown should state repeatedly that it was impossible to procure such bars. They will and can be found on every building constructed in this system of reinforcement.

In the tests in question the beams reinforced with the trussed bars actually did carry a greater load than those reinforced with the Johnson bars, and the fact that they did so was due to no other element than the rigidly connected diagonals, since the net area of steel in tension was practically the same in both cases. Comparing beams 1, 2, 3, Johnson, and 6, 7, 8, 9, Kahn, the only beams in the test that can be compared, the average load carried by Johnson beams was 11,753 lbs.; the average on Kahn beams was 12,737.

The tensional value of concrete is remarkably low, and probably in the neighborhood of 125 lbs. per square inch. The majority of the beams with straight line reinforcement failed by diagonal tension at the ends, and it would seem that even without the use of steel for shear reinforcement, these beams should be of sufficient strength to carry very nearly the same load as that at which they failed. The beams were so proportioned that with a load of about 12,000 lbs. the elastic limit of the trussed bar steel was exceeded, producing a marked elongation of the steel and a correspondingly high compression in the top causing failure.

In this special instance it was therefore difficult to obtain the true value of the trussed bar reinforcement, since if the beams were properly reinforced considerably more steel could have been developed with the given quantity of concrete.

Theoretically, the beams should be so designed that, like the one-horse shay, they should fail at all points at the same time. If they failed at any one point, then the reinforcement should be concentrated to strengthen up this weak point. Beams should not be calculated for bending moment at the centre and designed

Mr. G. Kahn accordingly, if the prevailing method of failure is by diagonal tension at the ends.

If the prevailing method of failure is by diagonal tension, then it would seem that the only way of guarding against this failure would be to have the diagonal members pointing upwards and crossing the lines of diagonal rupture. The spacing of these diagonal members should be closer towards the end than at the centre, and their size and number should be increased.

These diagonal members should, furthermore, be in no event farther apart than the effective depth of the beam, and in all cases should extend to the top of the beam. (Now, in this special test it must be remembered that Mr. Brown used diagonal members too short to properly perform their functions, excepting in one case. Trussed bars can be procured with diagonals of any length up to 48 inches).

It is very true that the correct result can be accomplished with the use of plain bars, but in a long beam it would certainly take a great number and would require careful and accurate placing. To be sure that all diagonal members are properly placed in the beam, it would seem that the reinforcement should be shop-prepared on an economical basis, it should be standardized and be economical to manufacture. It is also very true that the bar might be so sheared that each diagonal member is larger or smaller in size than its neighboring one. This, however, can be accomplished equally well by using several bars with sheared members, and, in fact, in almost every design wherein trussed bars are used, it will be observed that the sheared members at the end of the beam are very much increased in number.

The writer would say that the same care and attention should be given to reinforced concrete work designed with trussed bars as is necessitated by work designed for plain bars, and that if such care is given and if the shear members are properly proportioned, there is no question but the work can be done by workmen of less skill, and consequently can be more economically performed, field work being greatly reduced, and, as a whole, it will be decidedly safer.

In conclusion the writer desires to thank Mr. Brown and the members of the Canadian Society of Civil Engineers for the opportunity given to discuss this matter.

Prof. Brown The author is sure the meeting has welcomed Mr. Kahn's discussion, and is glad that it was found possible for him to personally bring before the Society certain points in which he considers the author made statements about the Kahn bar, which are erroneous, and calculated to mislead readers of the paper, as to the adapta-

bility of this form of bar to economical reinforcing of concrete. Prof. Brown
His views may be summarized as follows:

(1) That some ignorance is shown on the part of the author as to how Kahn bars should be used so as to properly reinforce a beam under given loading conditions;

(2) That the statement, at the bottom of page 5, that Kahn bars could only be obtained in the usual form, *i.e.*, sheared to the centre, is quite erroneous, and that as a matter of fact the standard bar is one which is unsheared over the central third, and sheared over the end thirds;

(3) That the comparison of relative weights of metal used for diagonal reinforcing by wing and straight bars (p. 6) is put in such a way as to mislead;

(4) That such comparisons as are made between the carrying capacities of beams of various forms are wrongly prejudicial to the interests of the Kahn bar, in so far that much of the metal of these bars, *viz.*, that in the wings of the central third where there was no shear, was really useless and should not, therefore, be considered as assisting the beam; and that in some cases, *viz.*, those of the small beams, 8" deep and 6' 0" c. to c., reinforced with 2-½" rods, the wings being cut only 6" long did not reach well into the compression area of concrete, and hence could not be expected to fulfil their proper function.

It is hoped that is a fair summary of Mr. Kahn's criticisms.

Yes.

Mr. G. Kahn

Prof. Brown

The meeting will no doubt desire to hear what there is to be said in response to these severe criticisms, both upon the author's abuse of the Kahn rods and upon the conclusions drawn. When Mr. Kahn, of Toronto, who is here to-night, wrote the author a letter along the lines of his criticisms which you have heard to-night, the author felt sure he had not read a correspondence between the author and the president of the Kahn company, Mr. J. Kahn, of Detroit. The author therefore forwarded to him copies of this correspondence, along with an invitation which the Secretary of the Society had authorized, to come to this meeting and say what he deemed necessary. He has brought his views to your notice, feeling, no doubt, that even in view of the correspondence referred to, the misconceptions which, apparently from the paper, exist in the author's mind, must be as publicly purged from the proceedings of this Society as they have been circulated by the paper amongst its members. The author hopes to show in as few words as possible the conditions under which the Kahn bars were supplied for test, and the restrictions which were placed upon the author's wishes by the Kahn Company of Detroit in regard to the

Prof. Brown disposition of the reinforcing bars, length of diagonal wings, etc., *i.e.*, virtually in all the directions in which Mr. Kahn of Toronto has criticised the paper. There is no need to publish the correspondence in detail, but the author must quote in full the essential points. (The italics are inserted by the author.)

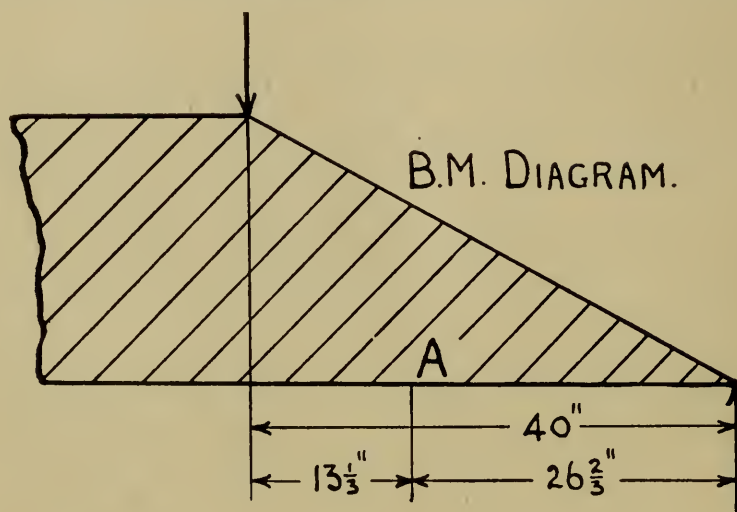
Extract from circular letter from Kahn Concrete Steel Co.—December 20, 1905.

"We will gladly supply any of your professors, instructors or students, the necessary Kahn reinforcing steel, free of charge, the only condition being that we may be informed as to the results of these tests, *and that the designs showing the placing of the reinforcing steel shall meet with our approval.*"

After discussing the tests with Mr. Goldmark, then of Montreal, the author was put into direct communication with Mr. Kahn.

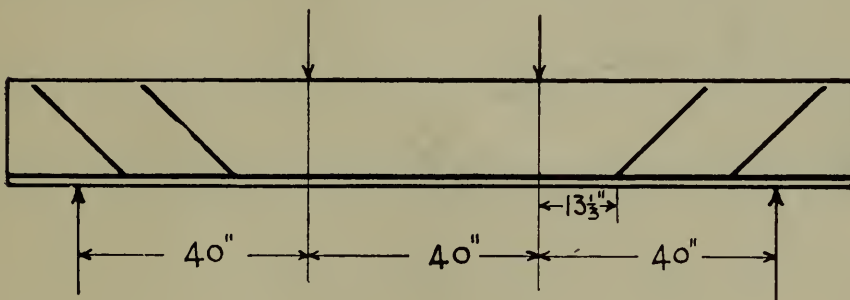
Extract from letter from author to J. Kahn, Esq., President of Kahn Company—Dec. 6, 1906.

"Take the case in point. I was proposing to use $2\frac{1}{2} \times 1\frac{1}{2}$ " bars giving section 0.76 sq. in. *unsheared* in a beam $8" \times 12"$ (outside)—



say $8" \times 11"$ above centre of reinforcement. The beams are 11 feet long, and will be tested on 10' centres. When sheared the net area of the bars will be 0.50 sq. ins. Hence the bending moment which the sheared bars would carry would be two-thirds that carried by the unsheared bar. The bending moment at A (see Fig.) is two-thirds the maximum, and A is about 27" from the support. There will be 6" overhang, making 33" in all to the end of the beam from

A. If the flanges are turned up at 45° the length would be about $11\sqrt{2}-16''$. This would give room for two wings at each end, and the bar would be somewhat as shown. There would be no diagonal reinforcement between A and the point of loading, but as A is approached the full value of the steel is not required for tension, and in any case with flanges bent at 45° there must always be some horizontal distance not covered by a diagonal bar. Unless this arrangement is such as would materially weaken the bond between the rods and the concrete owing to the reduced number of wings, it would compare very closely indeed with reinforcement with $3\frac{1}{2}''$



square bars (plain, twisted or corrugated), which we propose to test. These would give 0.75 sq. in. of steel against the Kahn 0.76 sq. in., and we propose to bend up the central bar at about the point A, leaving exactly the same amount of metal for tension as in the sheared Kahn bar, viz. 0.5 sq. in. If you consider that the above arrangement of the Kahn bars would permit of the full tensile value of the steel being developed, it would appear to be one which would harmonize almost exactly with our other tests as regards both percentage and disposition of reinforcing metal, and I would like to make the tests along these lines.

If, however, you prefer to shear the bars over the entire length, whether there is shear or not, it will be necessary to use a larger bar and figure on the reduced section.

J. Kahn to author.—Dec. 15, 1906.

"Your letter of 6th inst. has been referred to the writer. He has read very carefully your thorough discussion of this matter.

"Referring to your sketch it is very true that if the tension alone were considered in the main bar, the sketch such as you have shown would be quite in place. The fact of the matter is, however, that between the point of loading and A the shear is a maximum in just the same manner as it is beyond that point and near the support.

Prof. Brown "Our bars are fundamentally designed for a beam of uniform loading. Wherever special loading is desired, such as is proposed in your tests—that is where the load is applied at the third points—there is no question that the steel arrangement in the trapezoidal is along the correct lines. Our bars would therefore be somewhat at a disadvantage in such a case. *We would therefore suggest that our bars be sheared to the centre*, and that in making your calculations you allow only for the net section of the bar." Then follows a note that turning up a main bar would assist a beam materially, and a suggestion as to the advantage of using several small bars rather than one heavy bar such as the Kahn 1" bar. This point is noted in the author's paper.

Then followed letter from author to J. Kahn, Esq., December 17, 1906, asking for $\frac{1}{2}$ " bars for beams 6" wide, 8" deep, to test them using two bars in each beam against a single $\frac{3}{4}$ " bar. The author expected the $\frac{1}{2}$ " bars to be sheared so that diagonals would reach well up into the compression layer just as the diagonals of the $\frac{3}{4}$ " bar would, and in accordance with his suggestions in letter of December 6th in discussing the general question.

Letter from J. Kahn, Esq., to author.—Dec. 27, 1906.

"We will forward at once the bars you have requested. Our *Standard* $\frac{1}{2}$ " bars you will recall have diagonals 6" long, and the diagonals of the $\frac{3}{4}$ " bars are 12". If the $\frac{1}{2}$ " bars are used therefore in a deep beam they will serve very little better than ordinary deformed rods, because it is necessary that the diagonals extend well up beyond the neutral axis. . . . *These bars are all sheared to the centre.*"

(These points are fully noted at foot of table on p. 56.)

It is sufficient to leave this matter with the meeting. If the bars were used wrongly in the tests, it was due to the fact that the design submitted by the author, which is exactly what Mr. Kahn of Toronto says should have been used, was not accepted by his company. The author is not responsible for their views. *This is a matter of fact, and not of opinion.* It is claimed that the author's letters show:

(1) That he expressly asked for the identical form of bar which Mr. Kahn, in his criticisms, says should have been used;

(2) That after very careful reading of the author's letter of inquiry for these bars, the Kahn Company of Detroit elected to send him a different form of bar, sheared to the centre, and quite out of harmony with his own views;

(3) That the author's letter of December 6th shows that he was aware that diagonals should be specially sheared to reach well

above the neutral axis. (The author does not understand why the $\frac{1}{2}$ " bars supplied for the 8" beams were not sheared so as to agree with this suggestion.); Prof. Brown

(4) That the letter of the Kahn Company of December 27 indicates that the standard $\frac{1}{2}$ " and $\frac{3}{4}$ " bars, which were the only ones used in the tests, are all sheared to the centre.

The meeting will surely agree with the author that he has strong grounds for the statement to which Mr. G. Kahn takes such exception, viz., that the Kahn bars were only obtainable sheared to the centre.

The right ideas on the problem involved in the tests seem to have resided in Canada. Mr. G. Kahn and the author seem to be very much in accord in this matter, and it is hoped he will take it up with his Detroit office. There is no question but that the bars might have been used more advantageously, and that the material in a large beam may be disposed to far greater advantage than is possible in a beam using only a single rod. At the same time let it be fully realized that the author was not ignorant of these possibilities, and that the error was not his.

We will now deal with certain criticisms of Mr. Kahn with respect to the paper. He says: "Mr. Brown, in his paper, further says, that when the trussed bar is used, fully one-third of the metal performs no other function than to take care of shear." This is a sweeping generalization, made by Mr. Kahn and not by the author, of a statement on p. 6, which, as the context shows clearly, has special reference to the bars actually used in the tests. The early part of the paper (pp. 3-6) deals exclusively with the materials actually used, and the author certainly had no intention of conveying the impression that the disposition of material there described was inevitably fixed in all cases in practice in the precise proportions of the tests. The analysis on p. 5, and the author's letters to the Kahn Company, which have been read to you in reply to other criticisms, dispel that idea, but if Mr. Kahn feels that the author's statement is open to such misconception as he suggests, the author most willingly emphasises its restriction to the tests described.

At the same time it is the author's firm conviction that the Kahn system gives an unnecessarily heavy diagonal reinforcement, even when applied, as Mr. Kahn has explained to-night, by the aid of the blackboard sketch. On p. 58 of the paper there is recorded a test on a beam reinforced with $2\frac{1}{2}$ " Ransome rods running straight through the beam, and no diagonal reinforcement whatever was provided. Those who advocate the necessity of heavy diagonal bars would have said that that beam *must* fail at the ends, when loaded at the third points, owing to the excessive shear

Prof. Brown stresses. But it did not—it failed at the centre—in the manner termed *ideal* by those advocating heavy diagonal reinforcing. The concrete was very good—it was mature before testing, being one year old, and this single test is not put forward to counteract views as to the necessity of *some* diagonal reinforcing. But having seen the efficacy of a single bent bar, such as the Johnson or Ransome, in preventing the development of diagonal cracking on the outer thirds of the beams tested, the beams having subsequently failed in the central third, and bearing in mind the above result, there is no doubt a medium course by which all adequate diagonal reinforcing may be obtained by using straight rods as explained on pp. 5 and 6 of the paper, and at a *very* small expenditure of metal.

Mr. Shearwood has drawn the author's attention to the fact that the table on p. 56 shows that although many of the Kahn beams there tabulated contain a greater weight of steel than the Johnson beams, their net weight is less. This seems to indicate the possibility of voids forming by reason of the aggregate engaging in the many prong-like projections which the bars offer. This action, if it occurs to any great extent, might materially affect the bond.

Mr. Kahn claims that a comparison of the results of Kahn test beams Nos. 6, 7, 8, 9, with those of Johnson test beams Nos. 1, 2, 3, indicate an advantage for the Kahn system. (See Table, p. 56.) The author does not think that a comparison is altogether just, and refrained from it in discussing the results in the paper, but as Mr. Kahn has claimed an advantage, certain points which he neglected should be discussed. In the first place Johnson beams Nos. 1 and 3 had one bar out of three bent up at 45° , and a considerable portion of the end thirds was left without diagonal protection. This was expressly intended in planning the tests, and end failure was not surprising. Beam No. 2 had the diagonal bar bent at a smaller angle (about 24°), so as to come out of the beam over the support, and give diagonal metal over the entire portion of the outer thirds beyond the point at which it was bent up. The result was to give a higher load, and failure in the central third. Hence the average of these three tests is decidedly prejudicial to the Johnson bar in any comparison. The tests include two at one month and one at seven weeks, the latter being one of the diagonal failures. Any advantage of maturing concrete was probably lost. The four Kahn tests include two at four weeks, and two at eight weeks. Hence the average of the latter should include any advantage due to maturing of the concrete. All these points were taken into consideration, and the author therefore refrained from making comparisons. Mr. Kahn has ventured to claim a distinct

advantage, however, and, granting that the above considerations Prof. Brown are all in favor of the results of the Kahn tests, the comparison may go further than Mr. Kahn takes it, and we may consider the additional weight of steel necessary to secure the $8\frac{1}{2}\%$ advantage, which is all that can be claimed even when the considerations mentioned above are neglected. Beams 8 and 9 had $1\frac{3}{4}$ " rod, and the weight of steel given in the table includes that of the wings of the central third, which amounts to about 0.9 lb. per foot, *i.e.*, 1.8 lb. altogether, the beams being six feet c. to c. Hence a reduction of steel weight to $(17.0 - 1.8) = 15.2$ lb., may be claimed on account of the useless wings. In beams Nos. 6 and 7, containing $2\frac{1}{2}$ " rods, the corresponding allowance would be 1.9 lbs., leaving 15.9 lb. as the weight of steel actually in use in the beams. The wings on these bars were not as long as they should have been, and a further allowance of doubtful amount may be claimed. The average of 15.2 lb. and 15.9 lb. gives 15.55 lbs. of steel performing some useful function in these four beams which Mr. Kahn has selected. The average weight of steel in the three Johnson beams is 11.73 lb. The Kahn beams therefore contain 3.82 lb. more steel on the average than the Johnson, *i.e.*, practically 33%. The advantage in actual load capacity is $8\frac{1}{2}\%$. As an offset against the conditions which have been outlined as being in favour of the average of the Kahn tests, it may be granted that the single bar in Beams 8 and 9 would probably not distribute the stress so well as a larger number of small rods, but this consideration does not arise in Beams 6 and 7, containing $2\frac{1}{2}$ " rods. Even if it be assumed that *all* the wing metal of Beams 6 and 7 was practically inactive owing to shortness of wings, and this would not be the case, since the 6" wings reached up to $4\frac{1}{4}$ " above the tension reinforcement, the weight of active steel in these beams would be reduced to 11.7 lb., a figure in practical agreement with the weight of steel in Johnson beams. This allowance is, of course, altogether too generous, and the former comparison is probably more justifiable. On several grounds, then, the comparison, when closely examined, does not show the Kahn beams to the advantage claimed, if the weight of steel and hence the actual cost of reinforcing is considered. These considerations are quite apart from the relative costs per ton of various bars.

Mr. Kahn has discussed at some length the general question of diagonal reinforcement, and states that, except in one case, the author used diagonal bars too short to properly perform their junction. The correspondence quoted shows clearly that the author asked for bars with wings long enough to reach to the top of the compression side of beam, and was supplied with so-called standard

Prof. Brown $\frac{1}{2}$ " and $\frac{3}{4}$ " bars, with diagonals not as long as those asked for. Here again the author's views were set aside by the Kahn Company. Due allowance has been made for these factors in the above comparison of results suggested by Mr. Kahn, and the author is still of the opinion that, in any case the diagonal reinforcing afforded by the Kahn bars as at present rolled is too heavy. Mr. Kahn asserts that the trussed bar is fundamentally advantageous for beams of uniform loading where its great shear resisting capacity comes *fully* into play? What does he mean by coming *fully* into play? Can he state, and support his statement by any worthy evidence, what is the stress in the diagonals under working conditions? The author does not believe any such evidence exists, and knows of no tests which show what is the stress in the diagonals of any system of reinforcing. Unless it can be demonstrated clearly that there exists in the diagonals under working conditions a stress of approximately the same value as exists in the tension reinforcement under these conditions, or under the assumed conditions of design, the author cannot agree with the statement that the metal of the diagonals is *fully* developed. If the diagonals are not subjected to a reasonable working stress, it would seem that there is an excess of metal so placed.

The author wishes to thank Mr. Kahn for having come to the meeting, as it is far more satisfactory to have this discussion publicly than by letter. In so far as Mr. Kahn feels that at any point in the paper the impression has been conveyed that the proportion of diagonal metal in all cases of reinforcement by Kahn bars in practice is inevitably and unalterably the same as in the particular beams the tests of which form the subject of the paper, the author states unreservedly that he had no such intention. At the same time he does not think that a careful reading of the paper would lead anyone to suppose that he had that impression in mind.

Mr. G. Kahn The speaker would like to draw attention to the sketches on the blackboard, which hardly convey the correct idea. The beam looks as though it was all filled with diagonals owing to the fact that it is a small sketch with many lines. The fact that concrete will get in around those prongs is proven beyond question. Beams have been broken up in many cases to prove this. In practical construction, as other writers have shown, the concrete is not dry as in the tests, and it will flow around those diagonals without fail.

Mr. Brown just now stated that it had not been proven definitely that diagonal bars do perform their full duty or develop as we say they do. At Wisconsin University about six months ago a test was published in their *Bulletin* No. 197, which brought out this point possibly more strongly than anything ever done before. They

took a number of Kahn bars from which they sheared the wings Mr. G. Kahn entirely the full length. Then they placed them in beams the same as shown in the lower sketch, and made the beams in that manner. They took another set and placed them in the beams with loose vertical stirrups of the same sectional area as the wings sheared from the bar.

They took another set and placed stirrups diagonally of the same section as those sheared from the bar, and still another set of the Kahn bars in their original form. The speaker has not the exact figures here, but will be pleased to contribute them. They are something like this: The beams with the loose vertical stirrups (in all cases the same Kahn metal was used) carried about 10% greater load than the beams without any stirrups whatever; the beams with the diagonal stirrups carried 7% greater load than the beams with the vertical stirrups, and the beams with the rigidly attached stirrups carried about 13% greater load than the other with diagonal stirrups. The total difference was something like 32% between the Kahn bar with rigidly attached diagonals and the Kahn bar with the wings sheared off.

That is not quite the point. The tests quoted show only Prof. Brown that one form of stirrup is better than none at all, and that certain forms of stirrup are better than others. Mr. Kahn says the material is *fully* developed. By fully developing a bar is meant producing in it a suitable working stress—say 12,000 to 16,000 lbs. per square inch. If the actual stress in the bar is not known, how can it be asserted that the material is fully developed? If a smaller wing would give sufficient diagonal reinforcing under given conditions, the stress would be more fully developed, and this would be a real economy. If there is any definite information on this point, the author should like to know where it may be obtained.

The only way we can conclude is that the percentage of the Mr. G. Kahn load in the one case can be reduced to the load carried by that diagonal when placed in that manner.

The tests quoted mean nothing whatever as to the actual stresses Mr. Brown in the stirrups used. They merely show that one form of stirrup is better than another, and throw no light on the question of the economy of the steel used in the stirrups.

We have listened with interest to the two sides of this question, Mr. Francis and Mr. Kahn is entitled to credit for his frank discussion. Both gentlemen have gone into the subject very carefully. This discussion reminds one of a story told about an ex-Minister at Ottawa who was once called upon to decide a question of grammar

Mr. Francis between a witness and a lawyer, and his answer ran something like this: "Both is right."

Mr. Morssen Very good results have been secured by using Kahn bars, but it is also true that very good results have been secured by using the plain bars with loose stirrups. Reinforced concrete should be so designed as to allow the concrete to set freely. Concrete shrinks in setting, and stiff reinforcing diagonals or other units would interfere with its free setting, thereby causing cracks in the body of the concrete. The speaker has seen some work in New York where Kahn bars were used. When the labourers left at noon the beams were about three-quarters filled up. After dinner work began again and the concrete was put into place by tamping. The blows which the stiff diagonals of the Kahn bars received were transmitted to the main bar at the bottom of the beam, where the concrete had already partially set.

It is difficult to keep Kahn bars an inch or three-quarters of an inch from the bottom of the beam, while by using stirrups to hang the bars on we may ensure their distance from the bottom. Loose stirrups, moreover, can be so shaped as to separate the bars from each other, with the result that the concrete can be better placed. It is not certain either that the extra cost of the Kahn bars is justified by the increased strength of the beam, and the work is certainly not made any easier by them. The speaker has seen long bars bent in shipping, with their diagonals twisted in all directions. As it was no easy matter to straighten them they were put in just as they were.

The speaker does not agree with the statement that a less experienced foreman or less skilful workman can be employed by using Kahn bars if one is anxious to get good results, but is sure that equally good results can be got from the use of loose bars with stirrups, and a considerable saving made in the cost owing to the difference in the price of the materials.

Mr. Fyshe Mr. T. M. FYSHE (A.M. Can. Soc. C.E.)—Professor Brown in his paper draws conclusions about designing with ultimate and working stresses. Of course as he has evolved a straight line formula there will not be any difference between using ultimate stresses, dividing the loads thus obtained by the required factor of safety, and using as working stresses, ultimate stresses divided by the factor of safety. Using a parabolic compression curve, however, considerable difference will be found between using ultimate stresses, dividing the loads thus obtained by the factor safety and working stresses.

The speaker agrees with Professor Brown's recommendation of the use of a straight line formula as being close enough for all practical purposes. He believes, however, that the most important

point to consider, is not so much the formulae, as the working stresses adopted. It is now almost universally conceded that the ultimate strength of any reinforced concrete beam depends directly on the elastic limit of the reinforcing steel, and the speaker thinks more attention should be paid to basing the allowable working stress for steel on its elastic limit. Ideas as to what this working stress for steel should be, vary considerably, though as a general rule 16,000 lbs. per square inch is assumed for medium steel bars. Some building laws, however, call for working stresses equal to one-third the elastic limit, for example San Francisco and St. Louis. This is certainly a step in the right direction, for, 16,000 lbs. per square inch as a working stress for medium steel having an elastic limit of 30,000 to 34,000 lbs. per square inch is too high. The speaker advocates a working stress for the steel in the same ratio to its elastic limit as the ratio of working stress for concrete to ultimate strength of concrete. As a working rule he would suggest 12,000 to 14,000 lbs. per square inch for medium steel, and 18,000 to 20,000 lbs. per square inch for high elastic limit steel. In these experiments, of course, the steel never seems to be subjected to anything near the above-mentioned stresses at the instant when the concrete in compression is subjected to a safe working stress, for the tensile strength of the concrete is quite appreciable, taking care of perhaps one-half of the tensile stresses in the beam. It must be borne in mind, however, that these beams were only loaded once, and that it is very bad practice to figure on this tensile strength of concrete, as in time, through repeated loadings and possible overloadings, this value will be entirely destroyed.

With regard to steel used for these tests, the speaker believes that it is clearly shown that the strength of the beams varied as the elastic limit of the reinforcing steel used. The Johnson or corrugated bars used, for some unknown reason, did not come up to the average of this material, as these bars are generally rolled of material having an average elastic limit of 55,000 lbs. per square inch. With regard to Kahn bars, the speaker's great objection to them, outside of the great difficulty of handling and placing them on construction work, is that the amount of shear reinforcement in any beam is dependent on the length of the beam. One might have two beams of the same length requiring say only one Kahn bar for reinforcement, subjected to greatly different shearing stresses due to different loading. Here the Kahn shear reinforcement would be the same in both cases, obviously wrong. Using stirrups, however, the amount of shear reinforcement can be varied with great ease to provide for any possible shearing stresses.

Mr. Fyshe

The speaker believes the most essential thing in reinforced concrete to-day is to provide a reinforcement as nearly fireproof as possible. All systems should make provision for placing steel—perhaps along the lines of shop-made structural steel work—so that it would have to be placed exactly according to designs or not at all, leaving as few chances for mistakes as possible.

Prof. Brown

In his remarks upon the success which has attended the adoption of continuity in reinforced concrete work, Mr. Shearwood expresses a hope that possibly steel constructors may be encouraged in an attempt to attain similar ends in their own line of work. Continuity of girders, double intersection trusses, and statically indeterminate frames have for a long time been regarded as types of construction to be avoided. The long span double intersection rivetted truss is, however, again making its appearance, nickel steel is coming to the fore as a new material in bridge construction, allowing much higher working stresses than are possible with mild steel and thus altering the economic proportion of steel structures, and it may be that a new set of problems, or rather an old set in a new light, are being thrust before bridge engineers; and that a scientific study of these may result in the adoption of methods of construction hitherto regarded with some suspicion.

Perhaps the author misses some point in Mr. Shearwood's remarks, but cannot agree with him as to the advantage of having a large difference between yield point and breaking stress in steel used for reinforcement of concrete. A reinforced beam is practically useless when the yield point of the steel is passed; and within the yield point the deformation of steel would be the same for the same stress intensity whether the steel has a high or a low yield point. Cracking of the concrete begins when the steel stress is well within even a low yield point; and one of the principal arguments in favor of a high yield point is the smaller section of steel necessary to develop the compressive strength of a given amount of concrete.

The provision of as large a surface of rod as possible for a given cross section—say by using a $1" \times \frac{1}{4}"$ instead of $\frac{1}{2}" \times \frac{1}{2}"$, thus giving a perimeter of $2\frac{1}{2}"$ against $2"$, would give a reduced bond stress between the rod and the concrete, and such considerations would be useful in the case of severe repeated loadings. The devotees of deformed bars giving mechanical bond would claim an advantage here, and comparative tests as between a plain bar $1" \times \frac{1}{4}"$ and $\frac{1}{2}"$ deformed bars might be of interest.

Prof. MacKay has raised an interesting point in connection with the theoretical location of the neutral axis before the beam is

cracked. The analysis is simple, and is presented in an appendix to the paper. It forms a very good method of checking the experimental location of the neutral line at small loads. Reference to the appendix will show the formula deduced, n being the ratio of the full depth of beam to the depth to the steel centre line. The full depth is taken because the concrete is assumed to be entirely free from cracks. Five different cases taken at random from the tests, representing three types of reinforcement, are worked out. The greatest difference between the theoretical and experimental position of the neutral line expressed as a percentage of the depth to the steel, is 5%, and in three cases the agreement is very close indeed. These results form an interesting check on the work itself and on the straight line theory which the author advocates strongly. A considerable change in the value of the modulus of elasticity of concrete would affect the position of the neutral line but slightly, as the term affected by the change amounts to about 15% to 20% of the numerator, and to about half that percentage of the denominator of the expression giving the location of the line. This fact proves once again the comparative unimportance of the exact value of the modulus in the calculations. The loads for which the experimental values of x given in the tables are calculated, are about one-tenth of the breaking loads, and hence well within even working loads. This ensures reasonable security in the assumption that the concrete is uncracked.

The analysis which Prof. MacKay presents of compression stresses in the concrete when the beam is cracked to varying extents is of much interest. It shows how the position of the neutral surface varies according to the extent of the cracking, and indicates that the neutral surface in a beam which is partly cracked at one section is probably nearer the outer compression layer of the concrete than at a section where the concrete is uncracked. In fact, the elevation of the neutral surface would be some sort of wavy line, having elevations and depressions corresponding respectively to cracked and uncracked lengths of beam. This idea, which in all probability expresses the nature of the action as a beam is breaking up, shows how complex the problem really is at such stages of loading.

Mr. Holgate expresses his regret that the beams were not made in a manner more nearly approaching commercial conditions, in that a fairly dry mixture requiring tamping was used, rather than a wet mixture easily poured; and also that the composition of the mixture is given in weights of dry materials rather than in volumes as in practice. He therefore considers that the results are not of

Prof. Brown such broad application as might be desired. There are other aspects of the matter, however, which are worthy of some careful consideration. The principal object of the tests was not to determine a set of practical rules or a set of constants, for the design of reinforced concrete beams of varying mixtures, say 1—2—5, 1—3—6, etc., and varying percentages of reinforcement, but a much broader one, viz., to determine to what extent the ordinary laws of flexure of homogeneous beams are true in a reinforced beam. Once this point is clearly determined, and the author claims that his tests have shown that a simple method of analysis is in close agreement with the results of tests, and may be used safely in design, the general principles established are applicable to beams of different mixtures, provided the compressive strength and physical properties of these be determined by tests much simpler than those of breaking a series of beams of any particular mixture. To attain this object as accurately as possible, as uniform a mixture as can be obtained is desirable. Hence the materials were weighed in the dry state, and water was not added in excess of the amount required to give a good mixture when tamped. The water added *remained in the moulds*, and did not escape in the way in which it does in practice if the forms are only roughly put together. Such escaping water must carry off much cementing material. In some cases a person standing beneath a floor on which concrete is being poured is made unpleasantly aware of the fact by the constant dripping of water or even grout. In other cases he may be unaware of the process going on overhead. If the percentage of water is going to make a very appreciable difference in the properties of the resulting floors, there must be many cases in practice in which great differences exist, although the assumed conditions are the same. At a matter of fact, however, it is not found that a concrete wet enough to pour differs very much from one which has been tamped, and the practice of pouring is not followed with the object of securing a very different concrete, but rather on account of its cheapness and greater adaptability to the character of the labour usually employed on concrete work. The laws of flexure of a homogeneous beam can be determined, for example, by extensometer tests on a steel beam. Once established, we may apply them with a confidence which experience justifies, and to which further experiment would give additional proof, to beams of cast iron, wrought iron, timber, slate, cement, and even plain concrete. The author does not, therefore, quite agree with Mr. Holgate as to the limited scope of the results, but believes that a safe and economical design may be carried out in any average mixture of concrete, the properties of which are known, by an application of the elementary

principles, the establishment of which was the main object of this Prof. Brown investigation. If reinforced concrete construction is to be elevated to a position in which it can be justly regarded as having a reasonable claim to be considered as a well recognised and reliable form of construction, its fundamental principles must be on as sound a basis as those of other branches of engineering construction, such as bridge work. And although, owing to conditions of practice, actualities may be more removed from paper calculations than in some other fields of work, it should not be necessary to have to carry out exhaustive tests on each different mixture of concrete, and each type of bar, as a necessary preliminary to the production of a sound design.

Mr. Holgate has drawn the attention of designers to a very important point, viz., the scientific grading of the amount of cement used in various parts of a structure to secure the greatest economy of design. A simple illustration in another field of work may emphasize this point. If a high brick wall is to be built in a very exposed position in which it has to sustain high wind pressures, the lower courses may be laid in a rich cement mortar capable of resisting the tensile stresses at the windward side. A coarser mortar may be substituted in the upper courses. If the wall were laid in coarse mortar, not possessing the tensile strength of cement mortar, a thicker wall would be necessary to ensure stability, entailing extra cost of material and labour, as against the small cost of the cement necessary in the former case.

The author would like to have more details regarding the design and method of loading the floor which was designed for 250 lbs. per sq. ft. for concrete at 30 days, and carried 1500 lbs. per sq. ft. after maturing for one year without showing any effects of overload. One always suspects continuity effects, or more probably arching effect in the load applied for testing. In many cases in which floor panels are loaded for test in practice, pig iron, bricks, bags of cement, etc., are stacked on top of the panel. A little patient work on such a system of loading would enable the lower parts of the load to be removed, leaving an arch of self-supporting material abutting on the main beams and neighbouring panel.

The author does not think the effects Mr. Holgate mentions can be due to maturity of concrete. It could hardly mature to such an extent. But the case can be tested quite independently of the concrete. The steel, at all events would not mature. If the bending moment M due to the load applied is calculated, and equated to $kAfd$, where A = total section of steel and f = yield point stress of steel, a value can be at once obtained for kd , the distance from the steel centre line to the centre of the compression stresses in the

Prof. Brown concrete. If the loading conditions are really as assumed, and no arching is present, kd must come within the limits of the beam. In many alleged test cases the centre of compression is well up amongst the bags of cement, or other load. This point can be easily settled if the data are available.

Mr. Morssen's account of fire tests during which the material was under load are of great practical interest. The author's tests were made with the beams unloaded, the object being to determine to what extent heat treatment might affect the properties of the beam. They indicate a remarkable movement of the neutral axis, and a considerable reduction in carrying capacity. It must be remembered, however, that the conditions were severe in many respects, and the results cannot be regarded as applicable in a quantitative sense to the effects of actual fire. They suggest the nature of such effects rather than their probable extent.

Mr. Fyshe draws attention to the importance of designing for certain working stresses in steel and concrete, and suggests that the steel stresses sometimes used are rather high in view of the yield point of the material. The study of the phenomena during the gradual destruction of a beam given on pp. 62-67 of the paper, shows the great difficulty which exists in making an attempt to forecast the condition of the beam under actual working loads. If the beam is designed for a working stress of 16,000 lbs. sq. in. in the steel, and concrete tension is neglected, the actual stress will be less than 16,000 lb. sq. in. if the beam is uncracked. How much less, it is impossible to say. If the beam gradually cracked under loading, the steel stress would be 16,000 lb. sq. in., but it cannot be known in practice whether hair cracks exist sufficient to destroy concrete tension. If the yield point stress be used, and the ultimate load as figured on this be divided by a factor of safety, the same uncertainty will exist with respect to the actual steel stress. In any case, however, we can feel reasonably sure that it does not exceed a certain value, and more than that it is impossible to assert with much confidence. In the author's opinion a sound design may be made by either method, but possibly the use of a working stress conforms more to the methods of design in other fields of work.

With respect to reinforcing by Kahn bars under the conditions Mr. Fyshe supposes: Uniform distributed load, and concentrated loads, the amount of shear reinforcement can be adjusted to the conditions, if the bending moment be provided against by the use of a number of bars, some small and some large. Over the central portion of the beam carrying the distributed load the small bars may be sheared to give the light reinforcement necessary, while

towards the end the large bars may be sheared and the small ones bent up at an angle. For the concentrated loading the heavy bars would be sheared at the centre, and would thus provide against the large shearing force. By using loose stirrups, however, as Mr. Fyshe says, they can be placed in the necessary number just where required, and this gives great flexibility to design. Prof. Brown

Mr. Francis has alluded to the careful grading of the amount of cement in concrete mixtures, a point to which Mr. Holgate drew attention, and the results quoted of Professor Talbot's tests indicate that much valuable information, more especially in connection with the strength of reinforced columns, may be gained by a complete study of this aspect of the question.

Mr. Holgate has laid down an excellent motto for all engaged in reinforced concrete work: Vigilance is the price of safety. The author believes a close observance of that good rule would mean more than many formulae.

In conclusion, the author would like to thank the members of the Society for the patient way in which they have listened to a rather long paper on a previous evening, and to this long discussion to-night. Anything he has done in bringing the subject before the Society has been amply repaid by your interest and appreciation.

ELECTRICAL SECTION MEETING.

A meeting of the Electrical Section was held Thursday, November 26, Mr. H. H. Vaughan in the chair. A paper, "Some Considerations in the Application of Low Pressure Turbines to Power Generation," by Mr. J. R. Bibbins, was read by the author and illustrated with lantern slides. The manuscript of this paper was retained by the author for revision; both the paper and the discussion will appear in the *Transactions*.

WRITTEN DISCUSSION ON MR. MONSARRAT'S PAPER.

(See Trans., Vol. XXII., Pt. 1.)

By R. F. Leslie (M. Can. Soc. C. E.)

The writer has read with much interest Mr. Monsarrat's valuable paper on the erection of the French River Bridge, Canadian Pacific Railway; also the discussion thereon. Mr. Leslie

The question of moving ends of large steel spans brought up in the discussion is, in the writer's opinion, a very important one; and one which perhaps cannot be considered to have yet been met by an entirely satisfactory solution.

44 *Written Discussion on Erection of French River Bridge*

Mr. Leslie

The writer has had some twenty years' experience of maintenance of large steel spans, mostly with roller ends; and although, when properly looked after, these fulfil their purpose—in the majority of cases in a more or less satisfactory manner—they do not always do so; and when neglected, almost invariably cease to work; the end then either sliding, or apparently remaining motionless.

If the latter condition actually does obtain, it is obvious that most undesirable results must follow in the shape of additional strains and distortion of the truss members.

In any case such strains must occur, in a greater or lesser degree, when the roller end is not moving freely, and it is impossible to determine the amount of damage which might ultimately result through such a condition of affairs.

One solution of the problem is to suspend the moveable end of the span on links, and the writer is of opinion that, in the case of such spans as the one under discussion, there is much to be said in favour of this method.

The frictional resistance remains practically uniform under all conditions, while the possibility of foreign bodies or dirt causing rollers to jam is avoided.

The writer was employed on the launching of a somewhat similar span to the one under reference in 1887, where the suspended end was used, and with satisfactory results up to the present time.

The truss ends in this case were carried on single steel pins hung from links.

This bridge was a double-track one, over the Hooghly River at Hooghly, in Bengal, India, constructed by Sir Bradford Leslie, K.C.I.E., and consisted of a central cantilever span of 350 feet carried on two piers in the centre of the river, and two shore spans of 450 feet each.

The cantilever was erected in place, but as in the case of the Canadian Pacific Railway bridge, the great depth of water rendered it impossible to erect falsework, and consequently both shore spans were erected on the adjoining masonry viaducts and hauled across after completion.

In the case of the Hooghly bridge, however, matters were further complicated by a heavy tidal flow, which reduced the time available for the operation of hauling across to a little over two hours.

The launching of the first span was accomplished in one hour and thirty minutes, and both spans were hauled across into position without a hitch.

The weight of the shore span was 1200 British tons (or about

Written Discussion on Erection of French River Bridge 45

60 tons more than in the case of the Canadian Pacific Railway (Mr. Leslie span), and the floating end was supported by timber trestle work erected on a steel scow or lighter, the whole being well guyed both up and down stream from moorings to control the alignment.

The shore end was carried on specially constructed small-wheeled trollies and running on six rails each.

The winding engine was, of course, located on the central cantilever span.

Once in position the outer ends of the spans were secured by chains, and allowed to take their bearing on the ends of the cantilever as the tide fell.

At the present moment the writer has nothing beyond his memory to refer to, but should any member care to hear further details regarding this bridge, or of the suspension links used on it, he should communicate with Sir Bradford Leslie, who is now in England, and would be pleased to furnish the information.

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DONATIONS.

"A Practical Treatise on Railway Curves and Location, for Young Engineers." By WM. F. SHUNK. Philadelphia, 1901.

Presented by J. ED. THIBEAULT.

"The Design of Typical Steel Railway Bridges." By W. CHASE THOMSON. New York, 1908.

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"Engineering News." 1885 to 1889 inclusive.

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"Irrigation: Its Principles and Practice as a Branch of Engineering." By SIR HANBURY BROWN. New York, 1907.

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"Practical Irrigation," its value and cost, with tables of comparative cost, reservoir dimensions and capacities, etc. By A. J. BOWIE, JR. New York, 1908.

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"Canada's Fertile Northland." By R. E. YOUNG. Ottawa, 1908.

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"Steel Construction," a practical treatise on the modern use of steel in the erection of fireproof buildings, and its applications to structural work in general. By EDWARD A. TUCKER. Chicago, 1908.

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"Georgian Bay Ship Canal Survey," report on the precise levelling, 1904-1907. Ottawa, 1908.

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"The Materials of Construction," a treatise for engineers on the strength of engineering materials. By J. B. JOHNSON. New York and London, 1906

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"Hydro-electric Practice," a practical manual of the development of water power, its conversion to electric energy, and its distant transmission. By H. A. E. C. VON SCHON. Philadelphia and London, 1908.

Presented by H. A. ICKE. *Duplicate presented by* J. B. LIPPINCOTT COMPANY.

BY PURCHASE.

"The Practical Design of Irrigation Works." By W. G. BLIGH. New York, 1907.

"The Chemistry and Technology of Mixed Paints." By MAX TOCH. New York, 1907.

"The Distribution of Gas." By WALTER HALE. London and New York, 1907.

"Building Construction" and superintendence. Part II., 7th ed. By F. E. KIDDER. New York, 1908.

"The Mechanical Handling of Material," being a treatise on the handling of coal, ore, timber, etc. By GEORGE F. ZIMMER. New York and London, 1905.

"Fuel, Water, and Gas Analysis," for steam users. By J. B. C. KERSHAW. New York, 1907.

"Waterworks Distribution," a practical guide to the laying out of systems of distributing mains. By J. A. MCPHERSON. 2nd ed. London and New York, 1907.

"Water Softening and Treatment," condensing plant, feed pumps and heaters for steam users and manufacturers. By WILLIAM H. BOOTH. New York, 1906.

"Aerodynamics," being the first vol. of a complete work on aerial flight. By F. W. LANCHESTER. New York, 1908.

"Tools for Machinists and Woodworkers," including modern instruments of measurement. By J. G. HORNER. New York, 1906.

"The Management of Electrical Machinery," a revised and enlarged edition of "The Practical Management of Dynamos and Motors." By F. B. CROCKER and S. S. WHEELER. 7th ed. New York, 1908.

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"Handbook of American Gas Engineering Practice." By M. NISBET-LATTA. New York, 1907.

"Ordinary Foundations," including the cofferdam process for piers. By C. E. FOWLER. 2nd ed., revised and enlarged. New York and London, 1907.

"A Practical Treatise on Hydraulic and Water Supply Engineering," relating to hydrology, hydro-dynamics, etc., in North America. By J. T. FANNING. New York, 1906.

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QUEBEC BATTLEFIELDS FUND.

The subscriptions received from the membership for the fund to convert the Quebec Battlefields into a National Park amounted to \$592.05, contributed by one hundred and thirty-five subscribers. The expenses for postage, stationery, etc., amounted to \$74.75, leaving a net amount of \$517.30. On June 30, a cheque for \$509.60 was forwarded to His Excellency, Lord Grey, and this was acknowledged by letter as follows:

Citadel, Quebec,
2nd July, 1908.

"Dear Sir:

I am desired by His Excellency to acknowledge with thanks the cheque for \$509.60, which you are kind enough to enclose on behalf of the Canadian Society of Civil Engineers, and which is being forwarded to Mr. Courtney, the Treasurer of the National Battlefields Association.

Kindly express Lord Grey's appreciation of the generous gift.

I am,

Professor C. H. McLeod,
413 Dorchester Street West,
Montreal.

Yours, etc.,
J. HANBURY-WILLIAMS."

An additional cheque for \$7.70 was forwarded to Sir J. M. Courtney, Treasurer of the Association, on the 19th of September, and acknowledged by letter as follows:

Ottawa, Canada,
21st September, 1908.

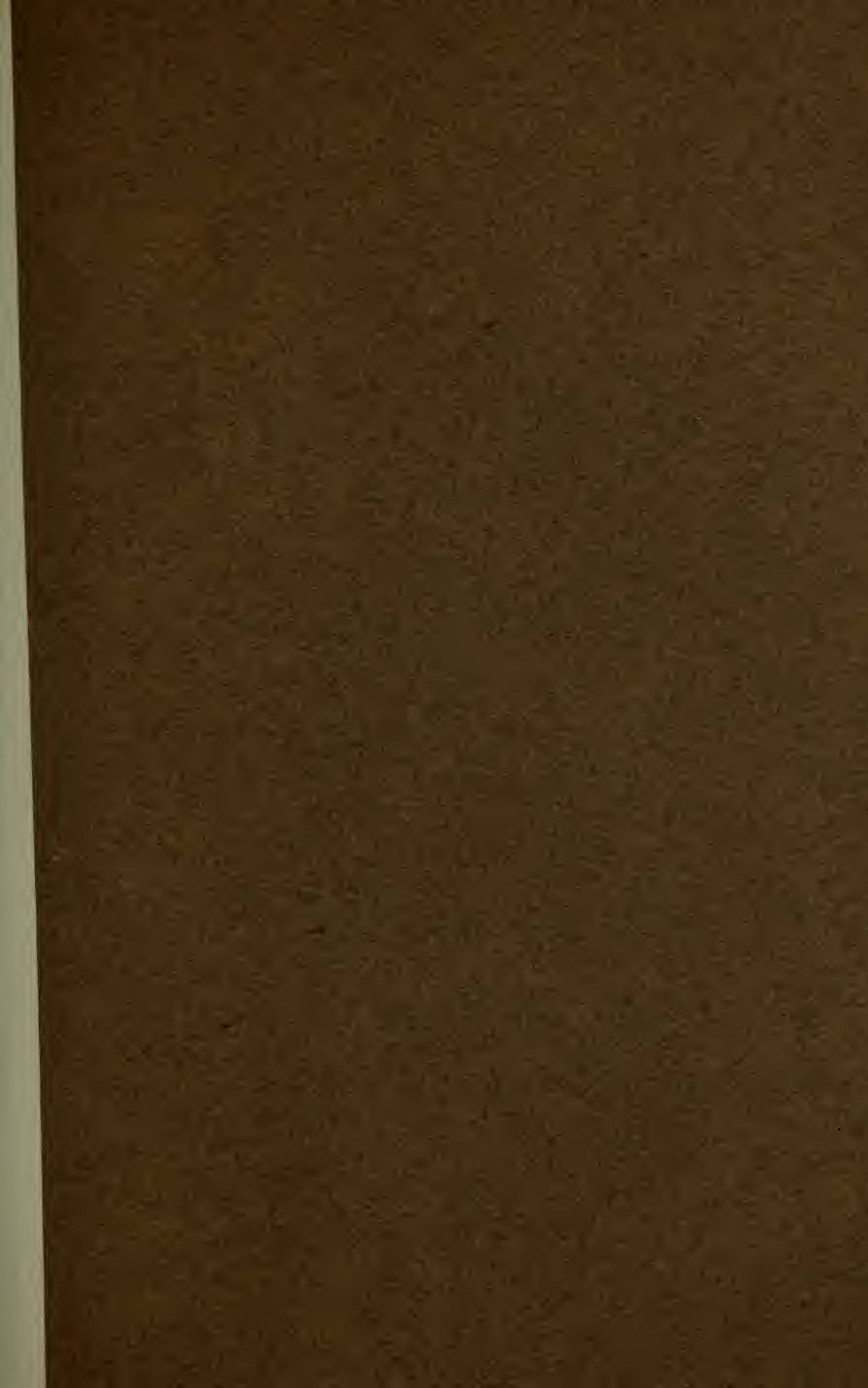
"My dear Sir:

I have your letter of the 19th with cheque for \$7.70, balance of contribution from members of your Society to the Battlefields fund, for which I have great pleasure in sending a receipt.

Yours very truly,

J. M. COURTNEY."

Professor C. H. McLeod.





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